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Probabilistic Seismic Damage Assessment of RC Buildings Based on Nonlinear Dynamic Analysis

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Abstract: The incremental dynamic analysis is a powerful tool for evaluating the seismic vulnerability and risk of buildings. It allows calculating the global damage of structures for different PGAs and representing this result by means of damage curves. Such curves are used by many methods to obtain seismic risk scenarios at urban level. Even if the use of this method in a probabilistic environment requires a relevant computational effort, it should be the reference method for seismic risk evaluation. In this article we propose to assess the seismic expected damage by using nonlinear dynamic analysis. We will obtain damage curves by means of the incremental dynamic analysis combined with the damage index of Park & Ang. The uncertainties related to the mechanical properties of the materials and the seismic action will be considered. The probabilistic damage curves obtained can be used to calculate not only seismic risk scenarios at urban level, but also to estimate economic losses.

Keywords: Damage curves, incremental dynamic analysis, uncertainties.

1. INTRODUCTION

For evaluating the seismic risk of existing structures, the damage grade has to be established for a given earthquake. There are several methodologies for calculating this damage grade; one of them is the Vulnerability Index Method [1-6] in which the action is defined by means of the European Macroseismic Scale, EMS-98 [7], by using macroseismic intensities and describing the seismic structural behaviour by means of a vulnerability index. Another widely invoked methodology is based on the capacity spectrum method developed by Freeman *et al*. [8] and Freeman [9] and further developed by Fajfar and Gaspersic [10], Chopra and Goel [11] and by Fajfar [12], among others. Although this approach is useful for large scale assessments, it can provide results which are not in agreement with those obtained by means of the incremental nonlinear dynamic analysis. The later allows calculating the global damage of structures for different PGAs and representing this result by means of damage curves. Such curves are used by many methods for obtaining seismic risk scenarios at urban level. In this article we use the algorithm proposed by Vamvatsikos and Cornell [13] to obtain probabilistic damage curves by considering as random variables the mechanical properties of the materials and the seismic action [14-17].

2. DESCRIPTION OF THE STUDIED BUILDING

The reinforced concrete framed building analyzed in this study, with 4 levels and 3 spans, has been designed and used in this article as a testbed for the proposed seismic vulnerability evaluation method; concrete framed building is shown in Fig. (**1a**) which also shows its geometrical dimensions. Due to its symmetry, the building can be modeled as bidimensional using a single frame (Fig. **1b**). The characteristics of the beams and columns of this frame are given in Table **1**.

The material of the beams and columns of the structural model follows an elastoplastic hysteretic rule with a post-yielding hardening of 5 %. Yielding surfaces are defined by the bending moment-axial load interaction diagram for columns and bending moment-curvature for beams. The applied dead and live loads follow the recommendations given by the Eurocode 2 [18] for reinforced concrete structures. The mechanical properties of concrete and steel are the values commonly used in the design of such buildings and the values are shown in Table **2**. Design standards require characteristic strength values for the materials obtained during the quality control process, from compression and tension tests in concrete and steel samples, respectively. By means of these tests, the concrete compressive strength, *fc*, and the elastic modulus of the steel, *Es*, can be modelled as random variables, a fact which is very useful due to the probabilistic approach applied in this article. Table **2** shows the mean, μ, the standard deviation, σ, and the coefficient of variation *cov* of these random variables and we assume that they follow a normal distribution. Other possible uncertainties, like those related to cracking and crushing of concrete, strain hardening and ultimate strength of steel, other effects such as the considering the slab in the model, axial force variations on column strength, just to name a few, can be also included in the probabilistic structural analysis, but in this article we consider only the uncertainties in the variables of Table **2**. In the following, we use the building described herein in order to explain the developments proposed in the article.

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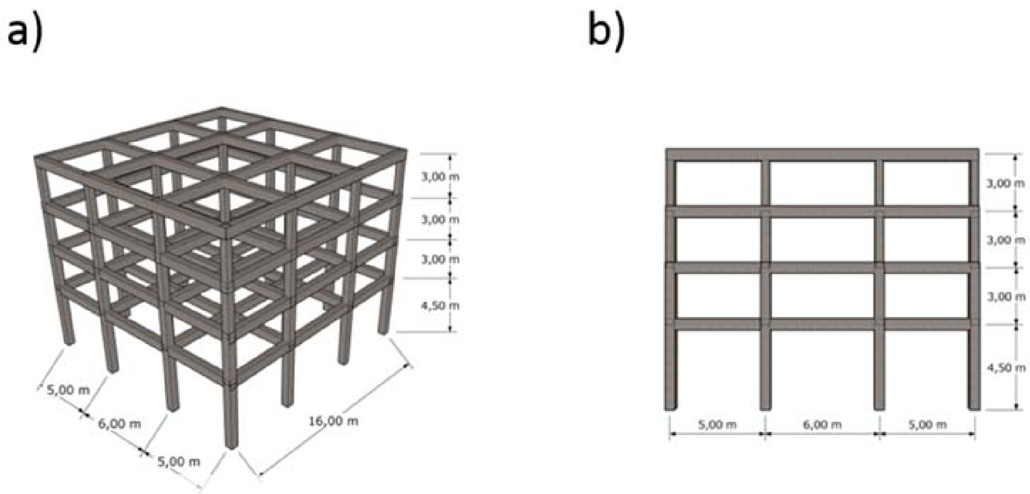


Fig. (1). **a**) One of the buildings studied in this article and **b**) 2D model.

Table 1. Characteristics of the elements of one of the studied buildings (Fig. 1). ***b*****,** ***h*** **and** ***ρ*** **denote base, height and steel quantity of the cross sections of the structural elements, respectively.**

	Columns			Beams		
Storey	*b* (m)	*h* (m)	*ρ*	*b* (m)	*h* (m)	*ρ*
1	0.5	0.5	0.03	0.45	0.6	0.0066
2	0.5	0.5	0.02	0.45	0.6	0.0066
3	0.45	0.45	0.015	0.45	0.6	0.0066
4	0.4	0.4	0.015	0.45	0.6	0.0066

Table 2. Characteristics of the input random variables. ***μ*****,** ***σ*** **and** ***cov*** **represent the mean, the standard deviation and the coefficient of variation of the input random variables.**

	μ	*σ*	cov
fc (kN)	2.1E04	2.1 E03	0.1
Es (kPa)	2 E08	2 E07	0.1

3. INCREMENTAL DYNAMIC ANALYSIS

A nonlinear dynamic analysis, NLDA, performed for a given accelerogram provides the time history response of a building and, then, the maximum response variables of the structure like the displacement at the roof, the global damage index according to a certain criterion, etc. can be calculated. Scaling the accelerogram with a given increment of the PGA, for values starting from a lower limit (which includes the elastic range) until reaching an upper one, corresponding to the building collapse and performing for each increment a NLDA, we can obtain a curve relating the PGAs to the maximum roof displacement, usually called dynamic push-over curve. When a curve relates the PGAs to the global damage index, it is denoted as damage curve. When, instead of a single accelerogram, several of them are used to perform nonlinear dynamic analyses and statistics are made with the obtained results, we are faced with an incremental dynamic analysis, IDA [13]. Summarizing, IDA allows obtaining the nonlinear dynamic response of a structure, for a group of earthquakes which are scaled to different measures of intensity which, in this article, is the Peak Ground Acceleration (PGA). Besides, this procedure has been extended to include uncertainties in the structural properties [19]. An important source of the uncertainty in the seismic response is the random variability of the ground-motion prediction whose influence has been studied by Bommer & Crowley [20]. The forecasting of the ground-motion parameters has been studied by Abrahamson *et al.* [21, 22], Bommer *et al.* [23] and Arroyo and Ordaz [24]. According to the probabilistic simulation approach used in this article, it is also necessary to describe the seismic action as a random variable. To do that, 10 earthquakes have been selected from the European database [25] and they can be seen in Fig. (**2**). This figure also

Table 3. Characteristics of the selected earthquakes.

Station Name	Date	Epicentre (Degrees)		Depth (km)	Magnitude (Ms)	Local Geology	Epicentral Distance (km)
		N	E				
San Rocco	15.09.1976	46.29	13.20	15	6.06	Stiff soil	17
San Rocco	15.09.1976	46.32	13.16	12	5.98	Stiff soil	17
Kotor Nas Rakit	24.05.1979	42.23	18.76	5	6.34	Rock	21
Auleta	23.11.1980	40.78	15.33	16	6.87	Rock	25
Ponte Corvo	07.05.1984	41.73	13.90	8	5.79	Rock	31
Matelica	26.09.1997	43.03	12.86	6	5.9	Rock	20
Tricarico	05.05.1990	40.65	15.92	12	5.6	Rock	20
Izmit-M-Istasyonu	13.09.1999	40.70	30.02	13	5.9	Stiff soil	13
Bolu-Bayindirlik	12.11.1999	40.76	31.14	14	7.3	Stiff soil	39
Athens-Papagos	07.09.1999	38.13	23.54	9	5.6	Rock	26

μ_{Depth} = 11 km, σ_{Depth} = 3.8 km, μ_{ED} = 22.9 km, σ_{ED}, = 7.65 km, μ_{Ms} = 6.13, σ_{Ms} = 0.55

shows the spectra of the selected earthquakes, their average spectrum, and the spectrum type 1 for a soil type A of Eurocode 8 (EC8) [26]. Fig. (**2**) shows that the procedure which has been used for selecting accelerograms compatibles with a seismic zone represented by a spectral shape allows obtaining a good fit. The list of these earthquakes and of their main characteristics is given in Table **3**.

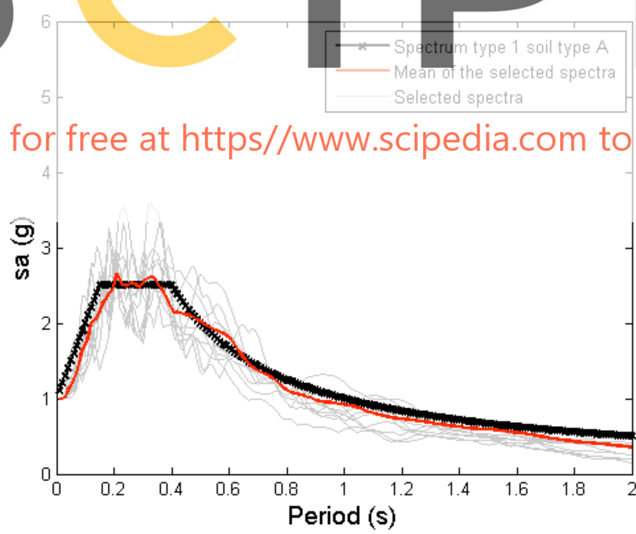


Fig. (2). Spectra calculated from the procedure based on the mean spectrum.

Several methods to establish the optimal number of accelerograms required to perform the inelastic dynamic analyses have been studied by Hancock *et al.* [27] and they conclude that the exact number depends on the damage measures which are considered and also on the predictability of the mentioned damage measures. Most of these methods are based on the magnitude and the spectral shape. Notwithstanding, we are not interested herein in obtaining an optimal number of accelerograms but a measure of the uncertainties in the structural response of the structures subjected to seismic actions. For this reason, in this article, the selection of the accelerograms is based on the difference between the mean of a group of response spectra and the target spectrum; the accelerograms corresponding to these spectra have been previously normalized to 1g. This difference is calculated based on the absolute cumulative difference between the target spectrum and the mean spectrum in several points; these points are separated by equal period intervals of 0.01s. The spectral shape selected in this study to the seismic action in the area corresponds to the type 1 of the EC8 which has a surface-wave magnitude greater than 5.5.

In order to include the uncertainties related to the structural characteristics, we use the Monte Carlo method. It is well known that the spatial variability between the characteristics of the structural elements greatly influences on the results [28, 29]. Therefore, we decided to consider it by generating, for all the columns of the same story of the building, one random sample for the compressive strength of concrete, *fc*, and, for each column of the same story, one random sample of the elastic modulus of the steel, *Es*. We used the same criterion to generate random samples for the characteristics of the materials of the beams of this story. It is important to note that the samples corresponding to each story are independent. This consideration is based on the fact that usually the structural elements of the same storey are made of the same concrete but the properties of the reinforcement can be supposed as independent from rebar to rebar. On the other hand, it is worth to recall that the objective of this article is the seismic risk assessment of an individual building; nevertheless, this objective could be extended to classes of structures existing in urban areas, taking into account, when modeling their risk, the building to building variation of the structural characteristics within a structural class. Further discussion of this issue can be found in Crowley *et al.* [30]. Afterwards, we generated random samples of the mechanical properties of the materials and performed nonlinear dynamic analyses, NLDA, for different accelerograms linearly scaled to values of the PGAs ranging from 0.1 g to 1.4 g at intervals of 0.1 g. We used the Latin Hypercube method [31] for generating random samples of the material properties and for combining these randomly with the accelerograms. Therefore, we performed 1000 NLDA for each PGA. The seismic global damage index of the building, which is necessary to calculate the damage curve, is obtained as a weighted mean of the damage indices initially defined at structural element level.

Several damage indices have been proposed for the elements of reinforced concrete structures starting from a post-process of the nonlinear dynamic response. Some of them are described in the following. A first simple method calculates the damage index as the ratio of the maximum ductility achieved during the seismic action to the ultimate ductility at element level and this is addressed herein as the ductility based damage index [32, 33].

$$\mathrm{DI}_E = \frac{\mu_m}{\mu_u} \tag{1}$$

where μ_m and μ_u are the maximum and ultimate ductilities, respectively, and the subscript E stays for element level damage index. Banon & Veneziano [34] proposed a damage index using a nonlinear equation considering the maximum and yielding ductility, the dissipated hysteretic energy, the yielding action and a numerator corresponding to monotonic loading.

$$\mathrm{DI}_E = \sqrt{\frac{\left(\frac{\mu_m}{\mu_y} - 1\right)^2 + \left(1.1\left(\frac{2E_h}{F_y\mu_y}\right)^{0.38}\right)^2}{\text{Numerator for monotonic load}}} \tag{2}$$

where μ_y is the yielding ductility, E_h is the hysteretic energy dissipated and F_y is the yielding action. The damage index of Park & Ang [35] is the sum of the maximum ductility divided by the ultimate ductility, that is, the ductility based damage index, with a term related to the dissipated energy. The corresponding equation is:

$$\mathrm{DI}_E = \frac{\mu_m}{\mu_u} + \frac{\beta E_h}{F_y \mu_u \delta_y} \tag{3}$$

where β is a non-negative parameter which represent the effect of cyclic loading on structural damage and δ_y is the yield displacement. Roufaiel and Meyer [36] proposed a damage index considering the maximum, the yielding and the ultimate ductility and, besides, the maximum and yield actions:

$$\mathrm{DI}_E = \frac{\frac{\mu_m}{F_m} - \frac{\mu_y}{F_y}}{\frac{\mu_u}{F_u} - \frac{\mu_y}{F_y}} \tag{4}$$

where F_m and F_u are the maximum and the ultimate actions, respectively. Bracci *et al.* [37] proposed a damage index as the ratio of the work done at the maximum ductility to the work done at the ultimate ductility

$$\mathrm{DI}_E = \frac{E_m}{E_u} \tag{5}$$

where E_m and E_u are the work done at the maximum ductility and the work done at the ultimate ductility, respectively. Cosenza *et al.* [38] proposed a damage index as the ratio of the maximum ductility minus one to the ultimate ductility minus one

$$\mathrm{DI}_E = \frac{\mu_m - 1}{\mu_u - 1} \tag{6}$$

In all the cases, the global damage index of the structure, *DI*, is a weighted mean of the element damages, in which the weights are the ratio of the hysteretic energy dissipated by each element to the total hysteretic energy dissipated by the structure [39]:

$$\mathrm{DI} = \sum_i \lambda_i \mathrm{DI}_E \tag{7}$$

where *DI* is the dynamic analysis based global damage index of the structure, λ_i is the ratio of the dissipated hysteretic energy of an element E to the dissipated hysteretic energy of the entire structure. Fig. (**3**) shows the evolution with PGA of all these global damage indices of the building of Fig. (**1**), considering the uncertainties related to the mechanical properties of the materials and of the seismic action.

In Fig. (**3**), one can see the important differences among the calculated damage indices, not only in mean but also in scattering, when uncertainties are considered. In this article, based on the results obtained in Fig. (**3**), we decided to select a damage index considering both the effect of ductility based damage and the damage due to dissipated energy. Two of the considered damage indices comply with this condition: those of Banon and Veneziano Park and Ang [34, 35]. The values obtained for the global damage indices are similar but we believe that the Park & Ang index is more adequately to represent the probabilistic damage curves because it clearly allows identifying which is the contribution of the dissipated hysteretic energy to the global damage index. According to the original calibration of the damage index made by Park & Ang [39] a value of 0.4 indicates that the cost of repairing the structure exceeds the cost of completely replace it, while a damage index of 1 indicates collapse. For this reason, in this article, when the damage index of Park & Ang is higher than 1, its value is set to 1. This makes sense, because, for higher PGAs, the standard deviation of the damage index decreases, what indicates that for higher PGAs the collapse uncertainty reduces. In Fig. (**4**) we show the mean and the standard deviation of the damage curves based on the damage index of Park & Ang. Such curves are the base of many procedures to estimate the seismic expected damage in urban areas.

CONCLUSION

In this work the risk of a framed reinforced concrete building has been assessed taking into account that the input variables are random. Not only the compressive strength of the concrete and the elastic modulus of the steel have been treated as random variables, but also the seismic action has been considered in a stochastic way. The approach to evaluate the expected damage of the building is based on the incremental dynamic analysis.

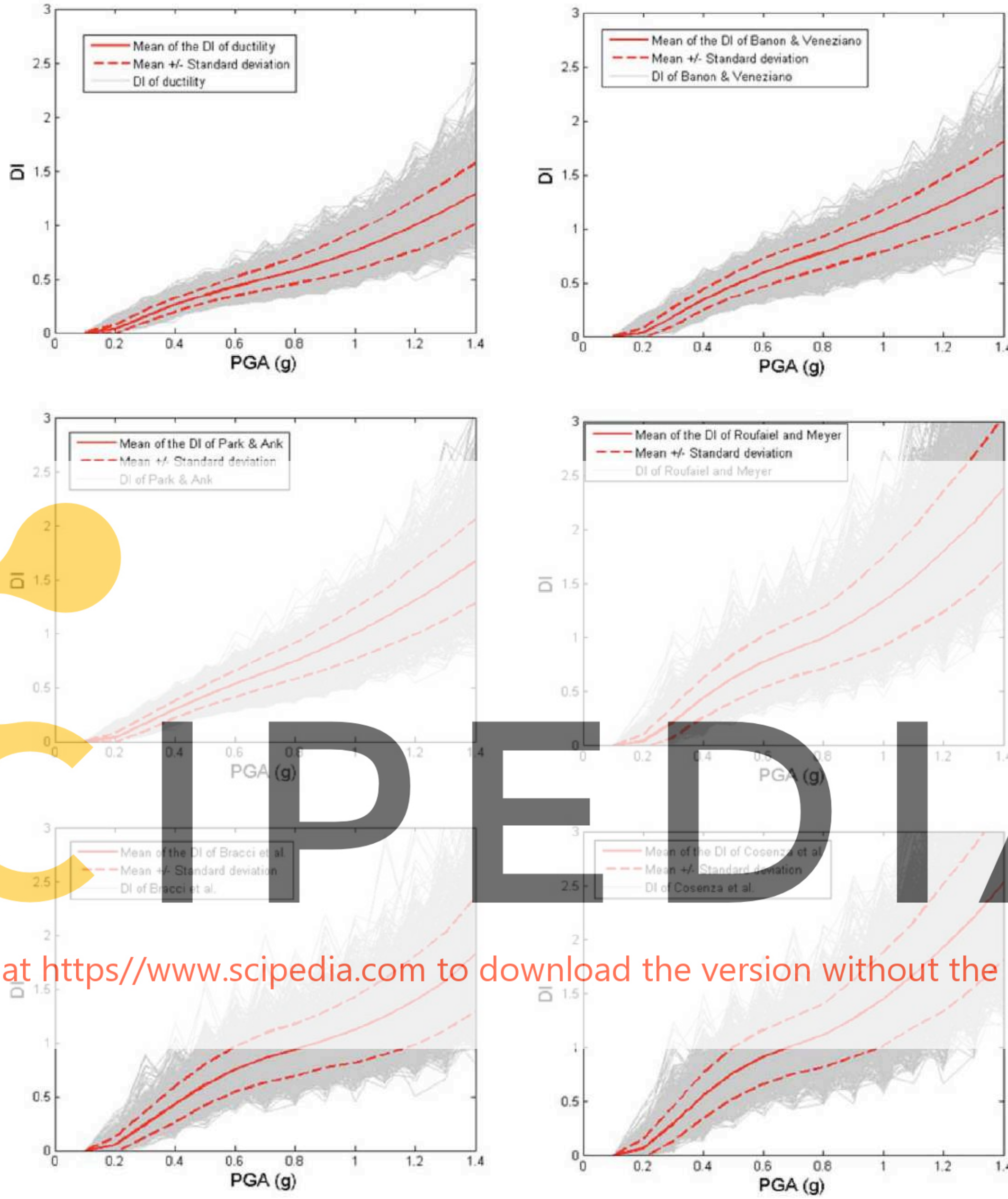
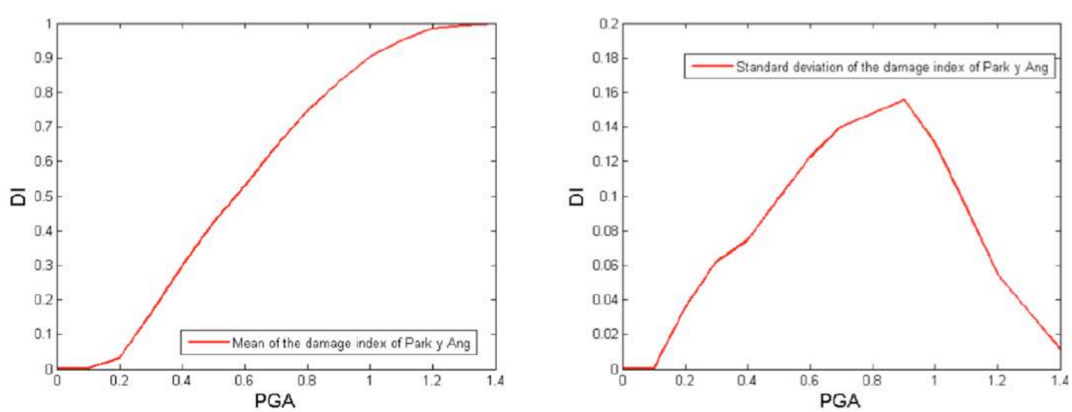


Fig. (3). Graphics of the discussed global damage indices represented in function of the PGA, calculated by means of non-linear dynamic analyses for the building of Fig. (**1**).



a) b)

Fig. (4). a) Mean and **b**) standard deviation of the damage index of Park & Ang.

An important conclusion is that, despite working with advanced non-linear structural analysis methods, the results show significant uncertainties when taking into account the randomness of the input variables. For this reason, the parameters influencing upon the seismic damage curves of the structures must be considered as random. Simplified deterministic procedures based on characteristic values usually leads to conservative results but some abridged assumptions on the definition of the seismic actions and on the estimation of the seismic damage states and thresholds can lead also to underestimate the real damage that can occur in a structure.

One of the major applications of the results obtained in this article is that one can estimates probabilistic seismic risk scenarios, based on non-linear dynamic analyses, because the required probabilistic damage curves defined in a parametric way, by means of their mean and their standard deviation, are available now.

CONFLICT OF INTEREST

The authors confirm that this article content has no conflict of interest.

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